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COAL ASH AS STRUCTURAL FILL WITH SPECIAL REFERENCE TO ONTARIO EXPERIENCE

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ABSTRACT

Fly ash and bottom ash obtained from coal-fired generating stations can be used as alternatives to natural materials for the construction of structural fills. The relevant engineering properties of coal ash are discussed in the context to their expected behaviour in structural fills. Four case studies of fly ash used as structural fill are presented to demonstrate that the physical behaviour of fly ash is similar to that of silt and that it can be handled with similar methods. The performance of fly ash structural fills were evaluated during and after construction. Groundwater monitoring data from existing fly ash fills are presented to show that ash leachate migrating in the groundwater regime has minimum impact on groundwater quality. Data is presented which shows that corrosion of metal services in ash should not be a concern. Ash leachate was found not to be detrimental to good quality concrete structures.

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Coal Ash as Structural Fill With Special Reference to Ontario Experience

1.0 INTRODUCTION

In order to produce steam in a coal-fired thermal generation system, the coal is ground by pulverizers into a fine powder and blown into the boiler where it burns like a gas flame. The heat produces steam used to drive turbine generators. Two types of ash are produced: fly ash and bottom ash (Figures 1 and 2). The fly ash accounts for approximately 80% of the ash produced and, in Ontario, is collected by electrostatic precipitators which are 97 to 99 per cent efficient in removing this particulate material. The fly ash is stored dry in silos or sluiced into lagoons. The other fraction, called bottom ash, is a combination of heavier particulate matter and molten slag, which forms on the inner surface and bottom of the boiler. The bottom ash is removed from the inner boiler surfaces using high pressure jets and is then sluiced into holding bins or lagoons.

The data obtained by Berry et al (1983) shows that the largest Canadian producers of coal ash are Alberta (2,027,000 tonnes) and Ontario (1,334,000 tonnes) which together represent almost 80% of the coal ash produced in Canada (4,321,000 tonnes) in 1983. Figure 3 shows the location and quantities of ash produced at the coal-fired thermal generating stations in Ontario during 1986.

The chemical, physical and engineering properties of ash are a function of: the type and source of coal; method and degree of coal preparation, cleaning and pulverization; type and operation of the power plant unit as well as ash collection, handling and storage methods. Ash from different sources has the potential to have a high degree of variability in its properties and may vary within the same boiler at various times in response to varying power demand.

The three types of coal burned in coal-fired generating stations are bituminous, sub-bituminous and lignite. The American classification of coal ash relates to the amount of CaO in the ash. The ashes with high amounts of CaO demonstrate self-hardening or pozzolanic properties and are called class "C" ash. Class "C" ash is typically obtained from lignite coal. The

ashes produced from bituminous and sub-bituminous coal are called class "F" ash and the ash does not possess pozzolanic properties. Canadians refer to the type and source of coal when describing the coal ash produced. For example, at Ontario Hydro's Nanticoke Generating Station, the ash is derived from a mixture of western Canadian sub-bituminous coal and West Virginia sub-bituminous coal. In the United Kingdom, the coal ash is called pulverized fuel ash (PFA). The pozzolanic properties of PFA have been documented by Mustard et. al. (1959) and Clendenning et. al. (1962).

Why Use Coal Ash as a Structural Fill?

The special properties and availability of coal ashes, both fly and bottom ash, render them quite suitable for use as a structural fill compared with natural soils and rocks in some situations. The low unit weight of fly ash allows its use for fills over soils with low preconsolidation pressures, such as alluvial clay or silt, where excessive unit weight could stress the soil beyond the preconsolidation pressure and may result in excessive settlement as well as possible failure of the subsoil. Coal ash may have a unit cost advantage in urban areas close to the source of ash production. Further advantages attributed to the use of fly ash are demonstrated when a project can be completed without the time delay penalty for stage construction or the expense for increasing the rate of consolidation through such measures as sand drains or surcharge loading, some of which may be required if traditional natural materials are selected.

Fly ash is easy to handle because it is light and does not bind into lumps requiring mechanical breakup. Fly ash will not stick to dump truck boxes or loader buckets like a cohesive material would. The ash can be placed and compacted using small compaction equipment in confined areas such as behind retaining walls and bridge abutments. In such situations, the reduced lateral stresses on the walls, resulting from the high frictional strength and low unit weight of fly ash, may yield savings in concrete quantities.

2.0 COAL ASH PROPERTIES AND BEHAVIOUR

Physical and engineering properties of interest include: particle shape and grain size distribution, moisture-density relations with respect to compaction, shear strength, compressibility, hydraulic conductivity, capillarity and frost susceptibility. In the following sections, the importance of the above items are discussed along with typical ranges of values cited from a number of sources. Reference is also made to a number of case studies in Ontario where measurements of properties or behaviour have been made.

Grain Size and Particle Shape

The grain size distribution is of interest because many engineering properties are related to the variation of particle sizes of the material. A summary of typical grain size analyses of bottom ash and fly ash produced at Ontario Hydro's Lakeview, Lambton and Nanticoke coal-fired generating stations are

shown in Figure 4. The range of gradings cover a 10-year period at Lakeview and a 5-year period at Lambton and Nanticoke. The range of gradings is relatively narrow at each station. As a measure of uniformity, the Hazen's uniformity coefficient, H_c , which compares the 60% passing size to the 10% passing size, shows for Nanticoke Generating Station fly ash a value of 16 and a value of 5 for bottom ash. Gradings for typical soils such as silty clay, silty sand and well graded silty sand and gravel are marked on Figure 4 for comparative purposes.

Fly ash is a silt sized material composed primarily of glass spheres together with some crystalline material and varying amounts of unburnt carbon in the form of coke. Bottom ash is typically grayish black in colour, subangular in shape and generally has a glassy texture. Some particles have a somewhat porous structure (Chan, 1983 a).

Compaction

Fly ash displays a variation of dry density with moisture content that is smaller than the variation from a more well graded soil with the same median grain size (Moulton, 1978). The tendency for fly ash to be much less sensitive to variations in moisture content than comparable natural soils could possibly be explained by the higher air void content of fly ash. Normal soils have between 1 and 5 percent air voids at maximum density; fly ash contains 5 to 15 percent air voids at maximum dry density (Moulton, 1978). The higher air void content could possibly tend to limit the build up of porewater pressures during compaction, thus allowing the fly ash to be compacted over a larger range of water contents.

Most natural soils experience an increase in dry density with an increase in applied compactive effort as shown in Table 1. The fine grained soils experience the greatest increase in maximum dry density and decrease in optimum water content of the soil types listed in Table 1. Fly ash, being a fine-grained material, would be expected to show similar behaviour. Ontario Hydro research indicates that fly ash does not experience density increases of the same magnitude with increasing compactive effort that other fine grained soils experience. Bottom ash, being a sand-sized material, has shown significant increase in maximum dry density and decrease in optimum water content, when compared to sand. The significant density increase may be caused by a partial crushing of some of the porous bottom ash particles under the higher energy input of the modified Proctor compaction test. Further discussion of the relationship between field compaction and laboratory standard tests, as well as a comparison of the accuracy of various field density measuring methods, will be discussed in Case Study 1: Booth Quarry Trial Fly Ash Embankment.

The behaviour of granular soils and bottom ash during compaction is different than that of fine-grained soils or fly ash. The addition of water to granular soils or bottom ash prior to compaction leads to the generation of surface tension forces which tend to pull the individual particles together, but which resist the rearrangement of particles in response to the compactive effort.

Examination of the moisture density relation would suggest that these materials be compacted either dry or at a water content very close to saturation (Moulton, 1978).

Specific Gravity

One parameter which influences soil density and is often used as a method of comparison for engineering materials is the specific gravity of solids. Higher maximum dry densities are achievable for ash of greater specific gravity. The specific gravity for fly ash ranges between 2.2 and 2.6 and for bottom ash ranges from 2.3 to 2.7 as shown in Table 2.

Shear Strength

The determination of the shear strength of the fly ash used for structural fill is necessary in order to evaluate the stability of any proposed slopes and the bearing capacity of foundations on the fill. The shear resistance of ash is dependent on such factors as: state of compaction, degree of saturation, loading and drainage conditions as well as the pozzolanic properties of the ash.

The ash derived from the bituminous and sub-bituminous coal that is burned in southern Ontario does not possess pozzolanic properties. Further discussion of ash properties and behaviour will be limited to the non-self-hardening ash. As southern Ontario ashes do not self harden, they display no true cohesion. Any apparent cohesive behaviour displayed will be lost upon complete drying or saturation and thus, cannot be counted on for design purposes.

Values of the friction angle obtained from various sources for fly ash and bottom ash are listed in Table 3. Values of friction angle of approximately 35 degrees will be noted for fly ash and a friction angle of 40 degrees for bottom ash.

The California Bearing Ratio test (CBR) is an ad hoc penetration test for the evaluation of subgrade strengths. The test results cannot be accurately related to any of the fundamental properties governing soil strength. However, the deformation of the soil specimen is predominantly shear deformation, thus CBR can be regarded as an indirect measure of shear strength. Several CBR tests were performed on Lakeview G.S. ash samples both in the "unsoaked" and "soaked" conditions. The test results are presented in Table 4.

Compressibility

The compressibility of a fly ash structural fill determines the rate and magnitude of settlement of structures founded on the fill. If the fly ash used in the construction of the structural fill is near the optimum water content or drier, then excess porewater pressures will not build up during and after compaction and settlement will not be time dependent. If lagoon ash with a water content that is much higher than optimum is used, then there may be some time dependent settlement as the excess porewater pressures dissipate.

The length of time for this is considerably shorter than for clays because the hydraulic conductivity of fly ash is at least one order of magnitude larger than that of clays. The compressibility of fly ash can be estimated in the laboratory using the oedometer. GAI Consultants Inc. (1985) report that the compression index, C_c , for virgin compression ranges from 0.05 to 0.37 and the recompression index, C_r , which is used for estimating the settlement from light to moderate loads on compacted structural fills, ranges from 0.006 to 0.04. Thus it can be seen that compaction can significantly reduce the compressibility of fly ash structural fills.

Hydraulic Conductivity

The hydraulic conductivity of fly ash is in the range of 10^{-6} to 10^{-9} m/sec. (GAI Consultants Inc., 1985). Materials in this range have poor drainage characteristics. Fly ash, because of its grain size, is particularly susceptible to capillary phenomena. Capillary rise is a concern because the fly ash can become saturated by groundwater which is drawn up into the compacted fill by capillary action. The ash will lose some of its strength and could become unstable. Capillary rise can be avoided by providing a drainage blanket of coarse material between the fly ash and the groundwater to intercept and drain any water before it can enter the fly ash fill.

Generally, the hydraulic conductivity of most compacted bottom ash is equivalent to that of sandy soils and it has good drainage characteristics. Values of hydraulic conductivity for ash are listed in Table 5. Capillary rise is not a problem in bottom ash because of its grain size. Bottom ash could serve as a drainage blanket below a fly ash structural fill.

Frost Susceptibility

The magnitude of frost heave is influenced by the ability of water to travel under osmotic pressure to the zone of freezing as it advances downward into the soil or ash.

According to the Ontario Ministry of Transportation and Communications (MTC) gradation requirements of granular "B" Type 1 material, for an acceptable subbase course material, the maximum permissible amount of passing No. 200 sieve particles is 8% of the total weight of material (MTC, 1982). This requirement is believed to be based on the potential frost susceptibility of the subbase material containing fines which are clay or silt.

A laboratory study was undertaken by Chan (1983 b) to measure the frost susceptibility of compacted bottom ash from Lakeview G.S. Specimens of bottom ash were compacted by the standard Proctor procedure and were tested under quick freeze conditions of -18°C for the air and $+4^{\circ}\text{C}$ for the water for either 141 hours (test series 1) or 311 hours (test series 2).

The frost heave results for the specimens are shown in Table 6. The frost heave measurements indicate that the compacted bottom ash is in the "satisfactory" classification by either the Reed classification (Reed, 1979)

or the T.R.R.L. classification (Jacobs, 1965) as modified for the different sample size. The amount of fine particles in the compacted bottom ash specimen did not have any significant effect on the amount of the heave. Both the granular "B" with 10% fines and the fly ash samples experienced unacceptable amounts of heave and would be classified as frost susceptible. The heave in the compacted fly ash was similar to that measured previously by Parg (1980).

The computer program THEAT was used to estimate the frost depth and by correlation between the modeled depth and measured depth, heat parameters were derived for the compacted fly ash trial embankment to be described in the Case Study 1. These parameters are as follows:

Thermal Conductivity,	frozen	0.341 BTU-ft/hr.ft ² . °F
	thawed	0.289 BTU-ft/hr.ft ² . °F
Heat Capacity,	frozen	16.5 BTU/ft ³ . °F
	thawed	22.8 BTU/ft ³ . °F
Latent Heat of Fusion	2000.0 BTU/ft ³	
Water Content	21%	

Comparable values for a typical fine grained soil are:

Thermal Conductivity,	frozen	1.092 BTU-ft/hr.ft ² . °F
	thawed	0.658 BTU-ft/hr.ft ² . °F
Heat Capacity,	frozen	20.0 BTU/ft ³ . °F
	thawed	33.0 BTU/ft ³ . °F
Latent Heat of Fusion	1500 BTU/ft ³	
Water Content	30%	

Therefore, because the conductivity of fly ash is considerably lower, and the heat capacity is only slightly lower, the frost depth in fly ash should be moderately less than in a typical fine grained soil, subjected to the same temperature.

In a typical structural fill, where the ash is not saturated and the freezing surface is several meters above the water table, measured surface heave was less than 8 mm (see Booth Quarry and Highway 402 cases). Frost depths ranged up to 1.0 m at these southern Ontario sites.

3.0 CASE STUDIES

Four case studies of fly ash being used as a structural fill in Ontario are presented to demonstrate that the physical behaviour of fly ash is similar to that of silt and that it can be handled with similar methods subject to certain minor modifications. The structural fill produced using fly ash will

perform as well as or better than the same fill constructed with silt sized natural materials with respect to ease of compaction as well as the amount of settlement of the fill and underlying soil. Table 7 will serve as a guide to the property or behaviour of the ash that is being emphasized in the following case studies.

Case Study 1

Booth Quarry Trial Fly Ash Embankment

The fly ash trial embankment was constructed to accomplish the following objectives: 1) to measure the density achieved by the field compaction equipment using several measurement methods and to compare the measurements on a statistical basis; 2) to measure frost penetration under winter conditions. Objective 2 has been previously discussed under frost susceptibility and will not be repeated here.

The embankment was constructed in a series of 0.2 m lifts to a completed size of 26.8m x 5.5m x 1.2m. The fly ash was trucked and end dumped at the site from covered dump trucks. A front-end loader was used to spread the material in lifts while a smooth drum, 30 tonne, 2000 rpm vibratory roller was used for compaction. All lifts were well compacted as indicated by the compactor riding up on the fly ash surface without marking it.

The fly ash used to construct the test embankment came directly from the dry handling facilities at Lakeview Generating Station. The hot ash was conditioned at Lakeview to a moisture content of 16 to 18% for handling purposes. No additional water was added during embankment construction. This fly ash has a typical silt size particle distribution curve. Standard Proctor density for the ash used was 1200 kg/m³ at an optimum moisture content of 31.7%.

Five methods of density measurement were used at various locations on each of the lifts during construction as follows: 1) nuclear density and moisture meter; 2) nuclear density with oven dry moisture determination; 3) Shelby tube sampling; 4) balloon volumeter; 5) one point Proctor method. A summary of the density measurements by method are shown as Table 8.

A statistical comparison of the overall mean value of dry density measurements by the student's method shows that only the mean densities of the balloon volumeter and tube sample can be treated as equal with a 95% degree of confidence. Of the two, it is recommended that the tube sample method be used in future tests of fly ash because of its relatively low variance and its conservative nature in that the sampling process shears the dense fly ash causing it to dilate as it enters the tube. This has been shown by Tavenas and LaRochelle (1972) to decrease the dry density by 15 to 50 Kg/m³.

The relatively large discrepancy between the nuclear density gauge measurement of density and other test methods can be partially explained by the fact that fly ash contains trace materials which absorb some of the thermal neutrons or

counts, which reflect the moisture content of the fly ash. This particular problem can be overcome through recalibration of the nuclear density gauge.

Density measurements at another site employed a nuclear density gauge that had been recalibrated for fly ash through testing in a compaction box. The overall variance of the recalibrated nuclear gauge was lower than the balloon volumeter, (private communication with W.J. Mason).

The one point Proctor test showed decreasing density with increasing moisture content for ash known to be dry of optimum, thus indicating that one point Proctors are an unreliable method of density measurement in fly ash.

The results of the mean dry densities obtained for a number of methods are shown plotted against the number of passes of the compactor in Figure 5. The figure shows that there is not a significant improvement in the density with further passes of the compactor after the first two passes.

Case Study 2

Highway 402 Interchange Embankment, Sarnia

Highway 402 is a four lane highway connecting Canada to the United States at Sarnia, Ontario. Construction of an overpass was required to eliminate a level crossing with Front Street - a major thoroughfare for the City of Sarnia - and a Canadian National Railway's spur line. This site is underlain by 30 m of compressible silty clay and layers of sand, totaling 7 m. Substantial differential settlements between the approach embankments supported on this clay and the pile supported bridge piers were expected if normal construction materials were used (Stuart and Selby, 1977). To avoid the costly and inconvenient alternative of staged construction, fly ash was selected as lightweight fill for the embankments.

Coal ash from Ontario Hydro's 2000 MW Lambton Generating Station was used in the overpass construction; 82,640 tonnes of bottom ash and 164,210 tonnes of fly ash. A plan of the overpass is shown in Figure 6 and a longitudinal section is shown in Figure 7. Separate embankments, 8 m high at the overpass, were constructed for the eastbound and westbound lanes.

Based on a subsoil investigation showing 2 to 3 m of compact sand overlying 30 m of stiff clay at the bridge site, an embankment settlement of 460 mm was calculated for normal construction materials (Stuart and Selby, 1977). Differential settlement was calculated to be 127 mm. Past experience with a similar structure 3 km east of this site indicated that actual settlements were about half of the calculated values using earth fill. With the lightweight fly ash fill used at this site, settlements in the order of 230 mm or less were expected.

As seen in Figure 7, the embankment design called for a lower layer of bottom ash, a fly ash core and an upper topping of bottom ash. The lower bottom ash layer acted as a drainage blanket for the fly ash and was also intended to

prevent the capillary rise of water into the fly ash core. Due to possible frost action in the silt-sized fly ash about 1.2 m of bottom ash was used to top the embankment. Bottom ash was used for the subbase material because it meets the MTC specifications for granular "B" Type 1 material. About 150 mm of granular "A" and asphalt were placed over the bottom ash subbase. A veneer of soil was placed on the embankment side slopes to prevent erosion of the coal ash and to support vegetation.

The ash fill was transported by tarpaulin covered dump trucks to prevent dusting problems during the 16 km haul from Lambton Generating Station to the site. Placing of the fill was accomplished with conventional earth moving equipment; bulldozers and smooth drum, self propelled, vibratory compactors. Proctor reference dry densities were 1587 kg/m^3 at 20.6% water content for the bottom ash and 1243 kg/m^3 at 30.5% water content for the fly ash.

Instruments were placed in the embankment to monitor:

- (a) settlement of natural soil below the embankment;
- (b) settlement of the embankment fill itself;
- (c) frost penetration; and
- (d) frost heave of the top of the embankment.

A complete set of instruments was placed at stations 123+00 and at 123+50 (Figure 5). The locations of instruments within the embankment are shown in cross section on Figure 8. Seven water manometer type settlement cells were placed in trenches below the embankment at each of the chainages to monitor settlement of the native soil during and after embankment construction. These cells were mounted on 300 mm square plates and connected to a remote gauge house.

Settlement of the embankment fill was monitored by settlement rods which were sleeved through the embankment and anchored in the underlying soil (Figure 9). Differential movement between the outer collar and the earth anchored rod was measured by dial gauge. Two settlement rods were installed at each chainage location.

Four frost heave and four frost depth gauges were installed, at each chainage, in the embankment shoulders. The frost depth sensors are plastic tubes filled with an indicator fluid that changes colour when frozen (Figure 10). Depth marks are scribed in the plastic so frost depth can be read directly. Sensors similar in concept to the settlement rods were used to monitor frost heave (Figure 11). A sight gauge forms the top of the heave sensor to allow direct readings.

During the January to April, 1982 monitoring period, frost depths ranging from 0.60 to 1.03 m were recorded. The instruments were located behind the guide rail and were often covered with snow. Frost penetration beneath the pavement surface would be moderately deeper than what was measured in the snow insulated shoulder area.

The frost heave sensors exhibited responses ranging from 5 mm settlement to 8 mm heave. In general, the settlement occurred on the north side of the embankment and the heave occurred on the south side. These results agreed well with a laboratory study by Chan (1983 b), that indicated that the underlying bottom ash was not frost susceptible.

Movement within the 8 m fill height was sensed by the settlement rods. Up to 2.45 mm of settlement was measured by these instruments - a negligible amount. The settlement of the underlying natural soil is more significant. Settlement cells installed in trenches beneath the embankment gave readings indicating the settlement of the native soil. After 10 months of monitoring, the average settlement across the two instrumented sections was 295 mm. Settlement readings show that 2/3 of the total movement had taken place about one month after fill placing was completed.

Groundwater sampling wells were installed at the locations shown in Figure 6. Groundwater samples were taken from the well locations on three occasions and major ion analyses were performed on these samples. As regional groundwater flow is approximately east to west, chemistry at wells 2 and 3 should represent background conditions. Well 1, near the embankment toe, showed no change over the monitoring period. Wells 4 and 5, downstream of the ramps, showed higher sulphate concentrations (~1200 mg/L) which may result from the ash fill.

Case Study 3

Shale Quarry Fly Ash Landfill

This 16 ha shale quarry, located in Mississauga, Ontario, is surrounded by commercial, industrial and residential areas. Quarrying has removed shale to depths of 25 m below grade, at which depth the high salt content of the shale makes it unsuitable. Since December, 1981 fly ash from Ontario Hydro's Lakeview Generating Station has been used to reclaim portions of the quarry for eventual use as a residential subdivision when quarry operations are complete.

During the filling of this shale quarry with fly ash, compacted as a structural fill, it was found that the densities being achieved in the field were below 95% of Standard Proctor. This degree of compaction was generally believed to be required to ensure adequate bearing capacity in a landfill environment. It appeared at the time that sufficient compaction was being achieved as evidenced by the riding up of both the compactor and loaded haul trucks on the compacted ash surface.

Short-term and long-term plate load tests were conducted on fly ash compacted to various degrees of Standard Proctor. The short duration tests used round plates of 0.3 and 0.6 m diameter. A 0.3 m square plate was used for the three month plate load test. Table 9 shows the results of the plate load test. Similar plate load test results have been reported by Leonards and Bailey (1982). All settlement for the long-term test occurred within the first hour

after load application.

On the basis of the good bearing capacity performance observed in the plate load tests, 90% of Standard Proctor was selected as the target density for the fly ash landfill (Cragg and Chan, 1985).

Case Study 4

Toronto Industrial Building Site

The building described in this case study was constructed on a coal ash fill that was originally designed for agricultural purposes and not as a structural fill. The geotechnical investigation by the owner's consultant is described along with the remedial measures undertaken to limit settlements to an acceptable level. In addition, the actual measured settlement of footings are described.

The site in Toronto, Ontario consists of about 20 ha of land, of which about 14 ha contain up to 12 m of coal ash. The fill, predominantly fly ash with some bottom ash, originated from Ontario Hydro's thermal plants in Toronto. The ash was placed prior to 1970 on a portion of Etobicoke Creek's flood plain that was not required to accommodate peak run-off conditions. The ash was filled to form a level surface over the original ground surface but the site was not compacted to the degree required for a structural fill. A subsequent owner of the site wished to construct an industrial building (305m x 60m) and locate it partially on the ash fill and partially on the native, competent, silty clay till.

The owner's consultant conducted a geotechnical investigation of the site. The investigation included borings into the ash fill and native till areas with split spoon and Shelby samples taken at each boring. Intact samples and prepared samples were subjected to triaxial and consolidation tests. In addition, two test pits were excavated to a depth of 5.8 m near the transition area between ash fill and competent till, to obtain large bulk samples for testing purposes and for in-situ visual observation of the coal ash.

Visual and textural classification of the samples retrieved from the borings indicate that the ash fill is predominantly fly ash with pockets or discontinuous layers of bottom ash. The ash fill is surfaced with a thin layer of silty clay 0.6 to 0.9 m thick, and is underlain by thin layers of alluvium on top of relatively competent shale bedrock.

There was an exceptionally wide variation in the standard penetration resistance, with N values ranging from 10 to 55 in the fly ash portion and up to 75 in the bottom ash portions. These values of penetration resistance agreed with those obtained by another consultant four years earlier, indicating that the coal ash had reached equilibrium condition with their present stress environment prior to that date. Friction angles of 35° to 36° for consolidated undrained triaxial tests were reported by both investigations.

Consolidation testing was performed on reworked and intact samples. The recompression index was less than 1.4 per cent in terms of strain and the compression index was 15 per cent in terms of strain. Large amounts of settlement could be avoided if the foundation pressures were kept below 200 kPa, which is the preconsolidation pressure. The footing design would not exceed this contact pressure.

In order to be assured that the preconsolidation pressure would not be exceeded under any of the footings and hence to avoid large settlement, the existing fill was excavated to a depth of 1.5 times the footing width in the coal ash area and to a depth of three times the footing width in the transition area as shown in Figure 12. The width of the footings were selected so that the footing contact pressure was maintained below 167 kPa. The expected settlements for footings founded on each of the areas are shown in Table 10. The total measured settlement for two of the footings founded in ash over ten months after the start of construction are 1.14 mm and 1.62 mm (Figure 13).

4.0 ENVIRONMENTAL CONCERNS REGARDING FLY ASH STRUCTURAL FILLS

Fly ash is not a natural material and as such, its interaction with the groundwater requires attention. Also, the concerns about the corrosion of some metals, which are used for underground services, and sulphate attack on concrete should be addressed. These three issues will be briefly discussed and experimental data will be presented to show that these environmental concerns are minor.

The highlights of the results from two Ontario Hydro fly ash landfill sites are presented here to indicate the extent of groundwater contamination resulting from the ash leachate. These landfills are not highway embankments, but it is believed that the degradation of groundwater quality by the ash would be similar in both situations. It should be borne in mind, however, that the extent of groundwater contamination by an ash fill depends on the hydrogeologic conditions of the site and the engineering design of the fill. The discussion of these factors is beyond the scope of this report.

The first site is Birchwood Park, which is an abandoned sand pit that has been rehabilitated, using fly ash as fill material during the period between 1968 and 1972 and is now part of the Mississauga parks system. The site was chosen for a detailed groundwater study in 1978 - 1980 by Ontario Hydro because the fly ash had been in place for ten years at that time and it was believed that sufficient time had elapsed for contaminant plumes to have fully developed in the groundwater system beneath the landfill. Further groundwater monitoring and studies were conducted at this site during the 1984 - 1985 period. The bituminous fly ash came from Ontario Hydro's Lakeview coal-fired station and it was used to construct a landfill in the form of a hill with about a 10% slope. The stratigraphy of the site comprises fly ash with a maximum thickness of about 17 m overlying a silt layer having a maximum thickness of 5 m. Underlying the silt is a 3-4 m thick layer of clayey silt. The movement of the contaminant plume is mainly in the more permeable silt layer.

At this site, a total of 40 piezometers have been installed for groundwater levels monitoring and for groundwater sampling. Chemical analyses were performed on the groundwater samples obtained over a period of several years. Table 11 shows a comparison of solute concentrations from Birchwood Park with the Guidelines for Canadian Drinking Water Quality (GCDWQ). An extensive hydrogeologic study of the site by Chan, Ash and Sykes (1986) has concluded the following:

"Major leachate components (sulphate, chloride and calcium) show elevated concentrations below and down gradient from the ash fill. Elevated concentrations of trace metals were confined to a shallow zone directly below the ash fill. The impact on the natural groundwater at the site has been small up to this point in time".

The second site is the shale quarry discussed in case study 3. A comprehensive environmental and engineering study of the ash disposal site was reported by Cragg and Chan (1985). Since the landfill operation was started, groundwater quality, surface runoff, dust, etc. had been monitored. The ash was placed in a number of "cells" in the quarry to facilitate the filling operation. Each "cell" was formed by a clay dike and an underdrain system was used to collect ash leachate which flows into a sump. Water samples were taken from this sump three or four times a year to study the quality of the diluted ash leachate. The results in Table 12 indicate that the quality of the natural groundwater is poor and that for most solutes the concentrations of the diluted ash leachate were within the Guidelines for Canadian Drinking Water Quality.

From these two Ontario Hydro landfill case studies, it may be concluded that the impact of fly ash fill would not be significant to the groundwater quality. Because the pH of Ontario Hydro's fly ash is high (approx. 9 to 10), only minute quantities of trace elements are leached from the material. With the effect of dilution in a groundwater system, the concentrations of sulphate, boron, calcium, etc. should generally be reduced sufficiently to meet the Guidelines for Drinking Water Quality.

The potential corrosivity of fly ash and bottom ash to metals such as those used in buried services could be a concern. This specific problem has been addressed through laboratory investigations by Headon and Chan (1982) and a field investigation by Headon and Chan (1985). The results of the 2-year field corrosion program are presented.

Table 13 gives the average corrosion rate of the specimens in mm/year. The first four metals are frequently used for underground services and stainless steel was used as a reference metal. The corrosion rate is an average of the rates of a triplicate set of samples and implies that all the mass loss has been due to a general corrosion and not to localized corrosion such as pitting. Table 14 gives a composite description of the characteristics of corrosion that has occurred on the triplicate samples. Some of the corrosion of samples has occurred in localized areas, mainly in the form of pitting,

which would have drastically different implications on different types of applications such as sheet piling or pressurized pipelines. The average corrosion rates for the five materials tested in fly ash and bottom ash are low and compare favorably with the average corrosion rates of the same materials in granular "A" exposed at the same site.

These results were obtained under conditions prevailing at the time of the experiments. The corrosion rates of these metals in Ontario Hydro's coal ash may vary slightly depending on the factors such as the moisture content in the ash, temperature, leachate composition, pH, etc. The results in Tables 13 and 14, however, would be good indications of the extent of metal corrosion in coal ash.

As indicated in Tables 11 and 12, the sulphate concentration in the groundwater near a fly ash fill may exceed 1000 mg/L. Therefore, the possibility of sulphate attack on concrete by fly ash leachate may be a concern to design engineers. As a conservative measure, a bituminous coating is sometimes placed on the surface of a concrete structure which is built adjacent to a fly ash fill. In some cases, sulphate resistant cement may also be used for the structure.

To address the potential sulphate problem, a laboratory project was undertaken by Ontario Hydro Research staff to study the extent of sulphate attack on cement mortar bars submerged separately in sodium sulphate solution and in fly ash leachate (approximately 2000 mg/L sulphate ion concentration). The results obtained by Hooton (1987) in the seven-year study have indicated that fly ash leachate with approximately 2000 mg/L sulphate ion concentration is not as aggressive* as an equivalent strength sodium sulphate solution to normal portland cements with up to 12% tricalcium aluminate contents.

From this laboratory experimental program, it may be concluded that the leachate from Ontario Hydro's fly ash should have little or no detrimental effects on good quality concrete structures.

5.0 SUMMARY

Large amounts of fly ash and bottom ash are produced at Ontario Hydro coal-fired generating stations. Bottom ash is quite suitable to be used in applications which would normally be satisfied with Ministry of Transportation and Communication granular "B" type 1 material. Fly ash has a low unit weight, thus is quite suitable for structural fills over soils with low preconsolidation pressures such as alluvial clays and silts. Fly ash can be easily handled with conventional equipment and compacted over a wide range of moisture contents. Typical friction angles are 35° for fly ash and 40° for bottom ash. Fly ash is frost susceptible if not properly drained.

*Based on the length of exposure required to reach either 0.05% or 0.10% expansion (considered detrimental), the sodium sulphate solution is between 2.1 and 2.4 times more aggressive than the fly ash leachate.

The case studies have demonstrated that the physical behaviour of fly ash is similar to that of silt and the structural fill produced with fly ash will perform as well as or better than the same fill constructed with silt sized natural materials, with respect to ease of compaction and to the amount of settlement of the fill and underlying soil.

From groundwater quality measurements taken at two of the case studies discussed, it was concluded that for most solutes, the concentrations of the diluted ash leachate were within the Guidelines for Canadian Drinking Water Quality. Because the pH of Ontario Hydro's fly ash is high (approx. 9 to 10), only minute quantities of trace elements are leached from the fly ash.

The corrosion of buried services in fly ash and bottom ash should not be a concern. Laboratory and field testing on samples of the most commonly used metals for buried services has shown that the corrosion rates of the materials tested are low and compare favourably with the average corrosion rates of the same materials in granular "A" exposed at the same site.

Sulphate attack on concrete was studied through a laboratory testing program. This experimental program concluded that the leachate from Ontario Hydro's fly ash should have little or no detrimental effect on good quality concrete structures.

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Table 1. The Influence of Compactive Effort
on the Maximum Dry Density and Optimum
Water Content of Soils and Coal Ash

Type of Soil	Average Results of Standard Proctor Test			Average Change Produced by Modified Proctor Compaction		
	Max. Dry Density (pcf)	kg/m ³	Optimum Moist. Content (%)	Max. Dry Density (pcf)	kg/m ³	Optimum Moist. Content (%)
Heavy Clay*	97	1554	28	+20	320	-10
Silty Clay*	104	1666	21	+17	272	- 9
Sandy Clay*	115	1842	14	+13	208	- 3
Sand*	121	1938	11	+ 9	144	- 2
Gravel-Sand- Clay Mixture*	129	2067	9	+ 8	128	- 1

*Tests conducted by the British Road Research Laboratory

Fly Ash** Lambton GS 1147 kg/m³ 35.6% +93 kg/m³ 9.8 %
 Bottom Ash** Lambton GS 1005 kg/m³ 28.3% +595 kg/m³ -11.0 %
 Bottom Ash Lakeview GS 1275 kg/m³ 28.5% +326*** kg/m³ -10.9 %

**Tests conducted by Ontario Hydro Research Division

***Transport Canada Data

Table 2. Specific Gravity for Coal Ash

Ash Type	Specific Gravity	Reference
Fly Ash	2.47	Clendenning et al (1975)
Fly Ash	2.4 ±0.16 (78 samples)	GAI Consultants Inc (1985)
Bottom Ash	2.66	Clendenning et al (1975)
Bottom Ash	2.5 ±0.17 (23 samples)	GAI Consultants Inc (1985)

Table 3. Values of Angle of Internal Friction for Coal Ash

Ash Type	Cohesion (kPa)	Friction Angle (Degrees)	Comments	References
Kanawha River Fly Ash	0	34.0	consolidated undrained triaxial on undisturbed samples	Pienpucta (1977)
Kanawha River Fly Ash	0	37.5 (as shown)	consolidated drained triaxial on undisturbed samples taken from a structural fill	Pienpucta (1977)
	0	32 (saturated)		
Fly Ash	0	33.1 ± 3.0	24 samples Eastern USA	GAI (1985)
Kanawha River	0	38.0	average void ratio 1.68	Seals et al (1972)
Fort Martin Unit 2 Bottom Ash*	0	40.0	average void ratio 1.41	Seals et al (1972)
Mitchell Bottom Ash*	0	42.5	average void ratio 1.08	Seals et al (1972)

*Direct shear on minus 9.5 mm material

Table 4. Comparison of CBR Values for Varied Compactions and Field and Lab Tests on Lakeview GS Ash

Material	Surcharge (g)	Layers/ Blows	Wet Density (kg/m ³)	Moisture %	Dry Density (kg/m ³)	CBR %
Field Fly Ash	11 339					6.7
Bottom Ash { unsoaked	9 071	3/25	1681	26.0	1334	22.0
soaked			1734	27.5	1360	15.3
Bottom Ash { unsoaked	9 071	3/40	1736	26.0	1378	24.1
soaked			1761	27.4	1382	23.8
Bottom Ash { unsoaked	9 071	3/55	1798	26.0	1427	36.4
soaked			1830	27.8	1432	26.5
Fly Ash { unsoaked	18 142	3/25	1652	32.2	1250	11.24
soaked			1690	41.1	1198	6.8
Fly Ash { unsoaked	18 142	3/40	1720	31.8	1305	15.4
soaked			1747	35.0	1294	13.5
Fly Ash { unsoaked	18 142	3/55	1740	32.4	1314	10.8
soaked			1740	34.1	1298	13.2

Table 5. Value of Hydraulic Conductivity for Coal Ash
and Some Soils Compacted to Maximum Dry Density

<u>Material</u>	<u>Dry Density (kg/m)</u>	<u>Void Ratio</u>	<u>Hydraulic Conductivity (cm/sec)</u>	<u>Reference</u>
Fly Ash			$1.9 \times 10^{-5} \pm 2.8 \times 10^{-5}$	GAI Consultants Inc (1985)
Fly Ash			10^{-4} to 10^{-7}	GAI Consultants Inc (1985)
Lakeview Bottom Ash	1451		4.8×10^{-3}	Chan (1983)
Lakeview Bottom Ash	1442		3.4×10^{-3}	Chan (1983)
Kanawha River Bottom Ash		1.05	5.0×10^{-3}	Moulton (1973)
Sands WS,SP			10^{-3}	Mitchell (1983)
Silts ML,MH			10^{-4} to 10^{-6}	Mitchell (1983)
Clays CL,CH			10^{-5} to 10^{-7}	Mitchell (1983)

Table 6. Laboratory Measurements of Coal Ash Frost Susceptibility

Specimen No	Material	% "fines"**	Water Content (%)	Dry Density (kg/m ³)	Measured Frost Heave (mm)	Remarks
1	Bottom ash	5	28.8	1265	0.54	The % fines in samples received from Lakeview GS varied from 4 to 5%
2	Bottom ash	5	24.7	1246	1.06	
3	Bottom ash	5	27.4	1264	0.83	
4	Bottom ash	9	26.5	1360	0.77	4% of "fines" was added to the view bottom ash to increase the % "fines" in the test specimens.
5	Bottom ash	9	27.4	1353	0.57	
6	Bottom ash	9	27.9	1354	0.60	
7	Bottom ash	12	27.9	1360	0.80	7% of "fines" was added to the Lakeview bottom ash to increase the total % "fines" in the test specimen
8	Bottom ash	12	27.8	1335	0.71	
9	Bottom ash	12	28.6	1376	0.63	
9	Bottom ash	12	28.6	1376	0.63	
10	Granular "B"	6	7.1	2061	1.71	Franceschini** Granular "B" material
11	Granular "B"	6	6.7	2023	3.83	
Specimens Used in Test Series 2 (311 Hour Test)						
1	Bottom ash	13	29.8	1352	1.56	Specimens were soaked for 1 day after they were set up in "Frost Box"
2	Bottom ash	13	29.5	1341	1.00	
3	Bottom ash	17	26.8	1427	1.95	
4	Bottom ash	17	28.0	1405	1.87	Specimens were <u>not</u> soaked in "Frost Box"
5	Bottom ash	13	29.1	1386	2.72	
6	Bottom ash	13	27.5	1389	2.41	
7	Bottom ash	17	26.4	1426	2.10	Lakeview fly ash
8	Bottom ash	17	26.4	1447	2.18	
9	Fly ash	83	27.7	1245	12.14	
10	Granular "B"	6	6.5	2086	2.56	Franceschini Granular "B" Caledon Granular "B"
11	Granular "B"	10	6.2	2036	21.96	

* Particles passing the No 200 sieve (0.076 mm openings)

** Supplier of granular material

Table 7. Property/Behaviour Emphasized by the Case Studies

Case Study	Property/Behaviour					Bearing Capacity
	Compaction	Density Testing	Frost Action	Settlement		
Booth Quarry	✓	✓				
Highway 402	✓		✓	✓		
Shale Quarry	✓	✓				✓
Toronto Industrial Building	✓			✓		✓

Table 8. Summary of Density Measurements by Method

Number	Number of Compactor Passes	Dry Density ₁		Troxler	Density ₂	Balloon Density		Tube Density	
		$\bar{X}$ (kg/m ³)	σ			Dry $\bar{X}$ (kg/m ³)	Density ₁ σ	Dry $\bar{X}$ (kg/m ³)	Density ₂ σ
1	2	1235	82.25	1305	75.76	1038	62.44	1106	62.77
2	2	1120	73.42	1240	21.42	972	10.14	1082	39.84
2	4	1147	28.31	1239	34.71	961	44.23	1090	28.36
2	8	1178	52.12	1270	67.22	1005	12.20	1099	27.69
3	4	1218	147.95	1337	179.89	-	-	1050	19.65
4	4	925	53.79	1043	33.16	-	-	-	-
5	4	971	25.27	1072	39.43	-	-	876	50.51
5	8	1077	28.18	1160	58.07	-	-	933	52.78
Overall Values		1109	112.13	1208	106.84	994	34.78	1034	91.53
Overall Variance(s)		11,001		9988		908		7181	
95% Confidence Limits		±43.0 kg/m ³		±42.5 kg/m ³		±22.9 kg/m ³		±34.7 kg/m ³	

NOTE: $\bar{X}$ = mean value σ = standard deviation 1 = oven dry moisture
2 = machine moisture

Table 9. Results of Plate Load Tests on Compacted Fly Ash

Test No	Plate Diameter (mm)	Fly Ash Density (% Std Proctor)	Plate Loading* Maximum Pressure (kPa)	Maximum Settlement (mm)
1	600	93.4	157	4.1
2	600	93.4	192	6.9
3	600	98.2	225	3.2
4	300	98.2	225	1.0
5	600	85.2	278	4.0
6	300	86.0	1246	7.3
7	300	100.2	1246	4.0
Long-term	300		240	1.0

Table 10. Summary of Footing Design

Area	Contact Pressure (kPa)	Expected Total (mm)	Settlements Differential (mm)	Depth of Coal Ash Compaction Required
Coal Ash	167	25.4	19.1	1.5B
Transition	167	12.7	12.7	3 B
Till	287	12.7	6.4	foundation on competent till

TABLE 11

COMPARISON OF SOLUTE CONCENTRATIONS FROM BIRCHWOOD PARK WITH
GUIDELINES FOR CANADIAN DRINKING WATER QUALITY (GCDWQ)

MAJOR IONS

SOLUTE	BENEATH THE CENTRE OF THE FILL	
	MAX CONC	
pH	6.5-8.5	9.7
Sulphate*	500	2540
Chloride*	250	386
Calcium*	10	556
Magnesium	150	115

TRACE ELEMENTS

SOLUTE	HIGHEST CONC. BIRCHWOOD PARK	
Barium	1.0	0.12
Iron*	0.3	0.42
Manganese*	0.05	2.6
Chromium	0.05	0.10
Copper	1.0	0.017
Zinc	5.0	0.030
Boron*	5.0	17
Arsenic	0.05	<0.010
Lead	0.05	0.026
Selenium	0.01	0.0071
Cadmium	0.005	0.0017

NOTE: All values have units of mg/L except pH

*Denotes solutes with concentration
exceeding GCDWQ

TABLE 12

COMPARISON OF SOLUTE CONCENTRATIONS FROM SHALE QUARRY
ASH LANDFILL WITH GUIDELINES FOR CANADIAN DRINKING
WATER QUALITY (CGDWQ) AND WITH BACKGROUND GROUND WATER QUALITY

Solute	1985 Data			CGDWQ	Background Ground Water Quality
Silver	<0.0002	<0.0005	0.0003		
Aluminum	<0.500	<1.00	<0.0200		
	<5.00	<5.00	<5.00		
	73.0	97.0	196		43 - 317
	<5.00	<5.00	<5.00		
Arsenic	<0.0010	0.0290	<0.0010	0.05	
Boron	4.51	5.17	3.30	5.0	2.1 - 3.4
Barium	-	<0.500	<1.00	1.0	
Calcium	246	240	170		75 - 2750
Cadmium	<0.0001	<0.0001	0.0002	0.005	
Chloride	409	455	363	250	185 - 26,400
Conductivity	303	284	264		
Chromium	<0.100	<0.200	0.0100	0.05	
Copper	0.0150	0.0020	0.0040	1.0	0.001 - 0.031
Iron	<0.200	<0.0500	<0.0500	0.3	0.1 - 0.26
Mercury	0.0002	<0.0002	<0.0002		
Potassium	66.6	50.4	52.0		50 - 250
Magnesium	31.2	27.8	38.0	150	30 - 877
Manganese	0.314	<0.0500	0.360	0.05	
Molybdenum	5.00	1.86	<0.500		<0.2
Sodium	406	400	280		100 - 11,400
Nitrate No ₃				10	1.3
Lead				0.05	0.002
	8.00	8.10	8.10	6.5 - 8.5	6.7 - 7.8
Selenium	0.0160	0.0410	0.0090	0.01	
Sulphate SO ₄	952	812	715	500	62 - 920
Strontium	5.95	7.00	4.30		
Titanium	<1.00	<1.00	<1.00		<0.01 - 0.08

NOTE: All values have the units of mg/L except Conductivity

*Denotes solutes with concentrations exceeding CGDWQ

TABLE 13

AVERAGE CORROSION RATE OF SPECIMENS
IN MILLIMETRES PER YEAR

ENVIRONMENT	CARBON STEEL	GALVANIZED STEEL	ALUMINUM 1100	1/2" TYPE M COPPER WIRE PIPE	STAINLESS 304
Fly Ash					
1 yr exposure	0.035	0.017	0.0004	0.0002	Too small
2 yr exposure	0.018	0.018	0.0006	0.0001	to measure
Bottom Ash					
1 yr exposure	0.031	0.004	0.002	0.001	Toosmall
2 yr exposure	0.023	0.014	0.001	0.001	to measure
Granular 'A'					
1 yr exposure	0.021	0.003	0.0004	0.001	Toosmall
2 yr exposure	0.011	0.003	0.0004	0.001	to measure
Corrosion Criteria (NACE, 1976)					
<0.03 mm/y		low			
0.03 - 0.13 mm/y		moderate			
0.13 - 0.25 mm/y		severe			
>0.25 mm/y		very severe			

TABLE 14

DESCRIPTION OF CORROSION AFTER TWO YEARS EXPOSURE
(A composite of three specimens)

ENVIRONMENT	CARBON STEEL	CALVANIZED STEEL	ALUMINUM 1100	1/2" (12.7 cm) COPPER WATER PIPE TYPE M	STAINLESS 304
Fly Ash	About 40% of the surface was corroded mainly in the form of pitting. The pits had a maximum diameter of 3.75 mm (0.15") with a maximum depth of 0.8 mm (0.03").	The corrosion was more severe on the lower face of the specimens. The black zinc-iron alloy layer was exposed on 50% of this face. 2% of the black zinc-iron alloy layer was exposed on the upper face. 0.05% bare base steel was exposed.	The specimens have corroded uniformly to a matte finish. This finish is a little less uniform on the lower face. Three micro-pits were found on one specimen.	Minor tarnishing of the surface.	No visible corrosion.
Bottom Ash	About 50% of the surface was corroded, mainly in the form of pitting. The pits had a maximum diameter of 1.6 mm (0.06") with a maximum depth of 0.5 mm (0.02").	The corrosion was more severe on the lower face of the specimens. The black zinc-iron alloy layer was exposed on 35% of this face. 0.1% of the black zinc-iron alloy layer was exposed on the upper face. No bare steel was exposed.	The corrosion occurs at isolated areas on a total of 5% of the surface. The corrosion is in the form of pitting or as etched areas containing pits. The largest pits are 1.6 mm (0.06") in diameter and 1 mm (0.04") in depth.	Overall micro-pitting of the surface.	No visible corrosion.
Granular 'A'	The corrosion was at isolated areas on about 15% of the surface of the specimen. There are pits within these areas up to 1 mm (0.04") diameter with a maximum depth of 0.25 mm (0.01").	No black zinc-iron alloy layer was exposed. Spangles (zinc crystals) are still visible. A grey coating has replaced the normal bright coating on 10% of the surface.	The corrosion occurs on a total of 3% of the surface as spots of slight tarnishing with blistering at a few locations. There are pits under the blistering. The largest pits are 1.5 mm (0.06") in diameter by 0.08 mm (0.03") deep.	Overall etching of the surface.	No visible corrosion.



FIGURE 1

Fly Ash and Bottom Ash

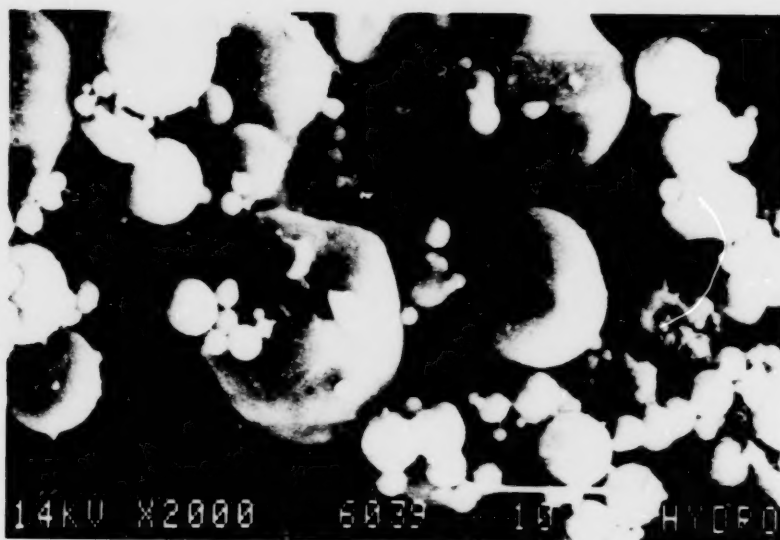


Figure 2(a)

Micrographs of Fly Ash at Various Magnifications
(Note: line shown in picture equals 10 micros)

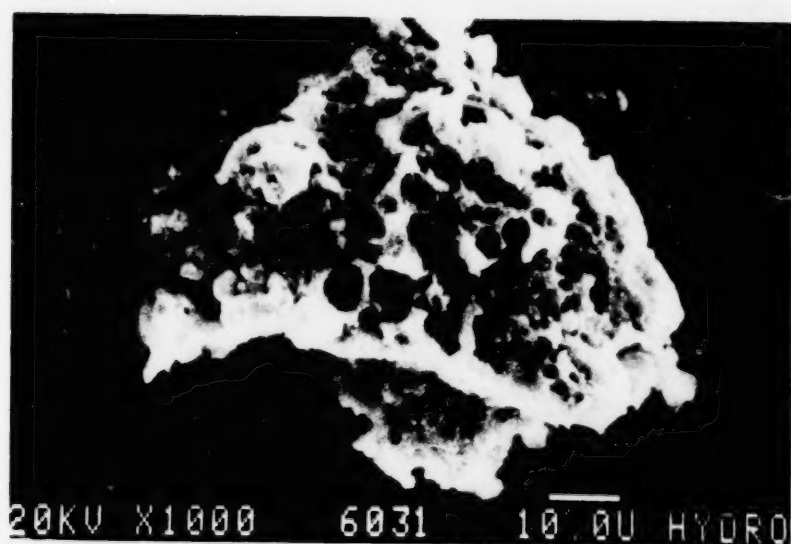
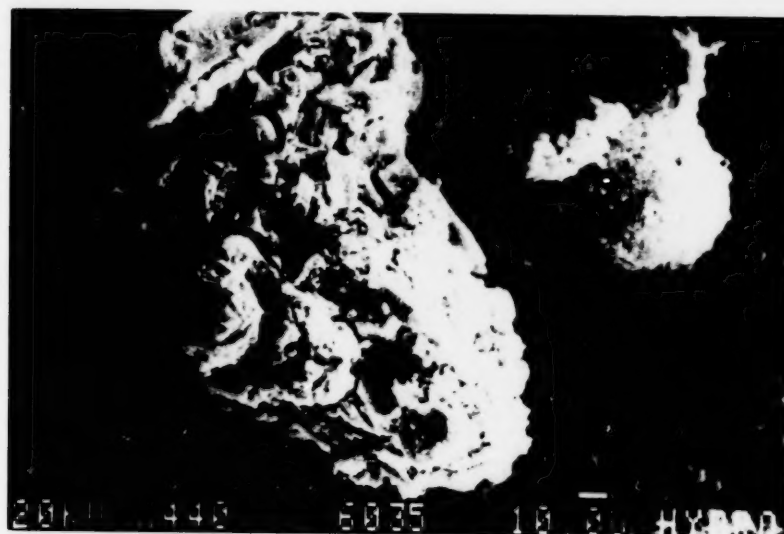
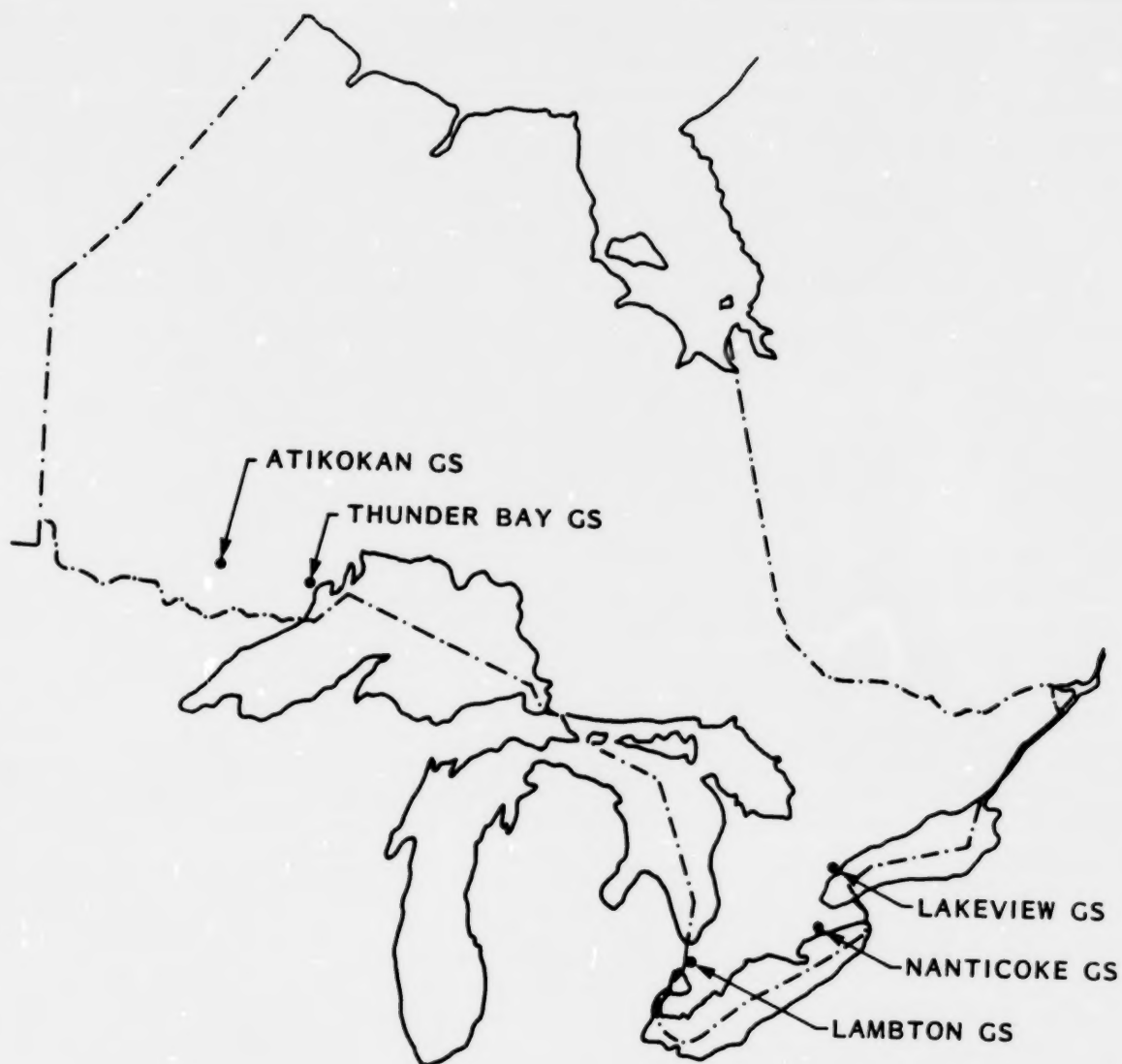


Figure 2(b)

Micrographs of Bottom Ash at Various Magnifications
(Note: line shown in picture equals 10 micros)



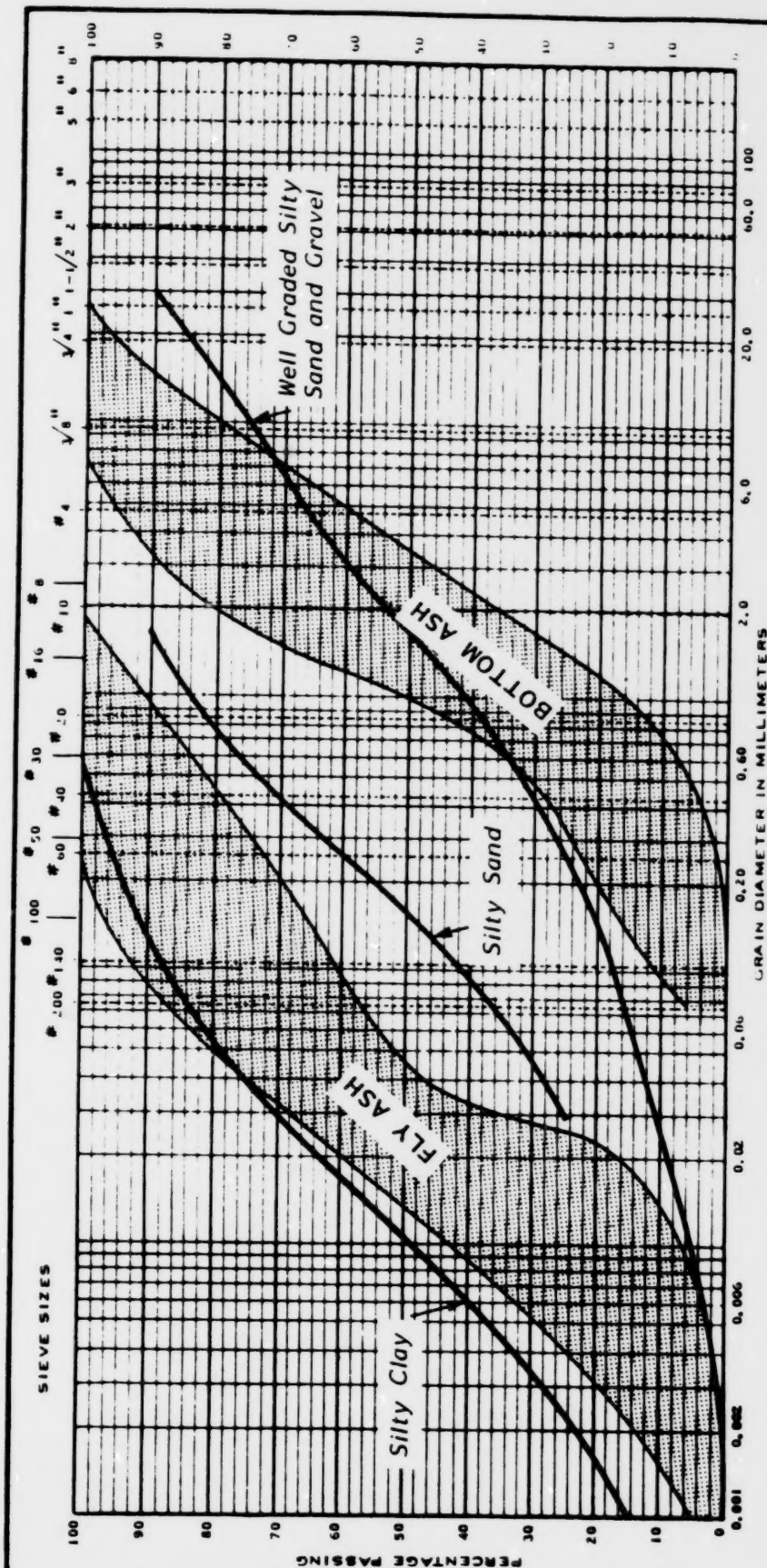
QUANTITY OF COAL ASH PRODUCED IN ONTARIO
BY ONTARIO HYDRO COAL-FIRED GENERATING STATIONS

GENERATING STATION	FLY ASH (TONNES)	BOTTOM ASH (TONNES)
LAKEVIEW GS	60,050	8,006
LAMBTON GS	193,250	25,767
NANTICOKE GS	336,175	44,684
THUNDER BAY GS }	57,037	29,332
ATIKOKAN GS }		
TOTAL	646,512	107,789

FIGURE 3
SOURCES OF COAL ASH IN ONTARIO

174913-RD

MECHANICAL ANALYSIS



TO SILT		SAND		GRAVEL		COBBLES	
CLAY		FINE	MEDIUM	COARSE	FINE	COARSE	

UNIFIED SOIL CLASSIFICATION



90992
new 80-11

Ontario Hydro
RESEARCH DIVISION

FIGURE 4
SUMMARY OF GRADING CURVES FROM
LAKEVIEW, LAMBTON AND NANTICOKE GS

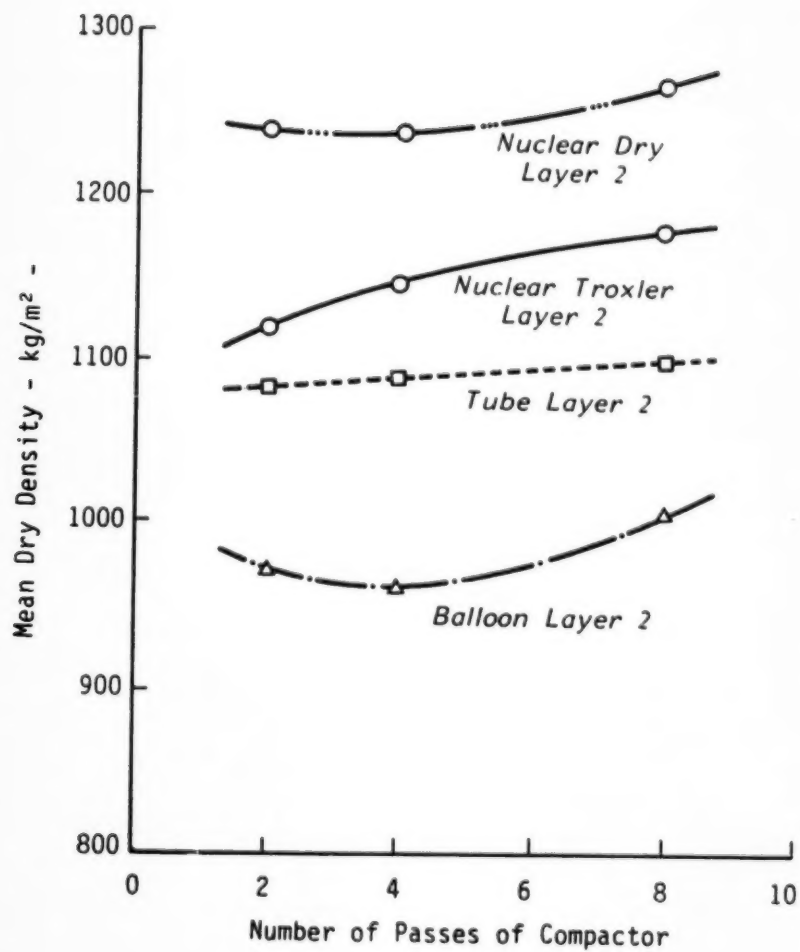


FIGURE 5
COMPARISON OF MEAN DRY DENSITIES WITH
NUMBER OF COMPACTION PASSES

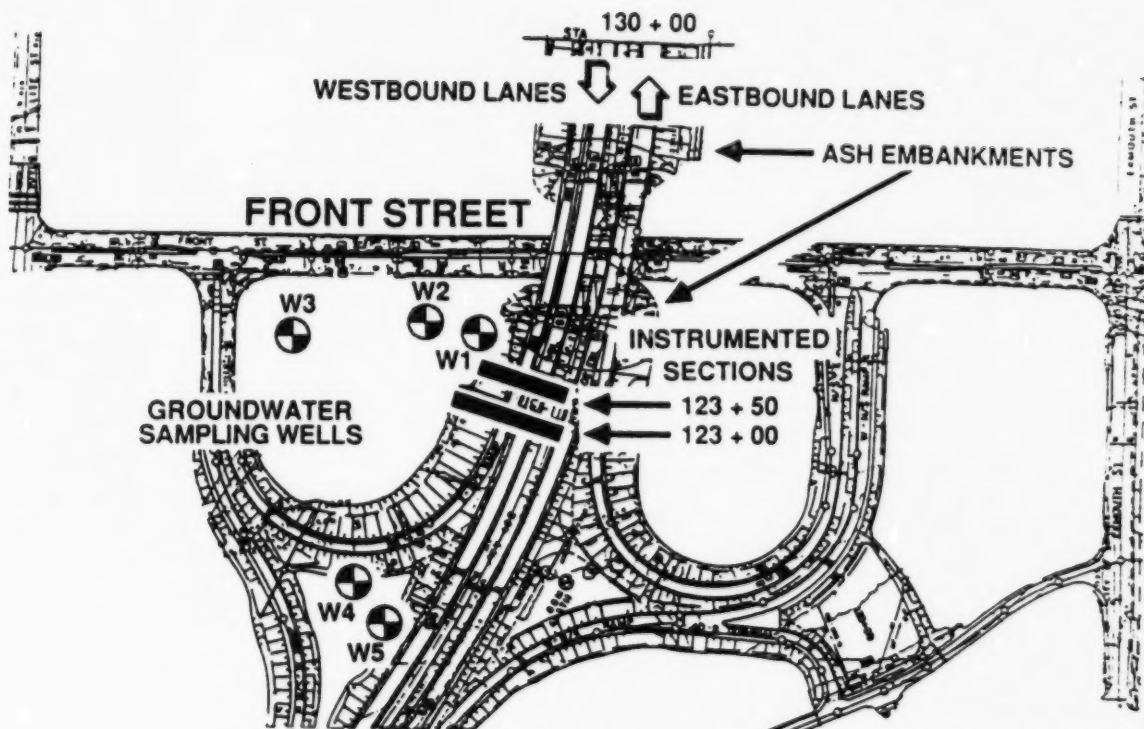


FIGURE 6
HIGHWAY 402 AT FRONT STREET, SARNIA
(From MTC Contract 81-47 Drawings)

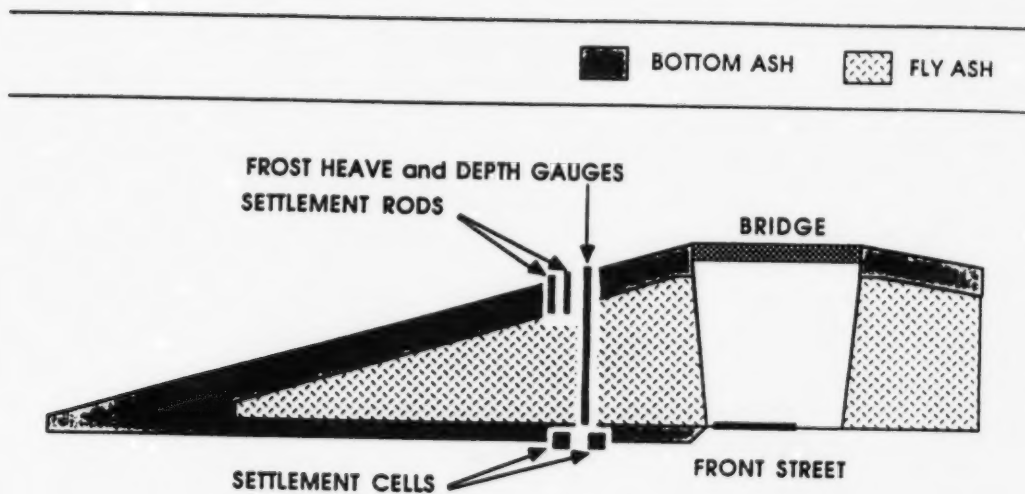


FIGURE 7
LONGITUDINAL SECTION OF HIGHWAY 402 OVERPASS
AT FRONT STREET

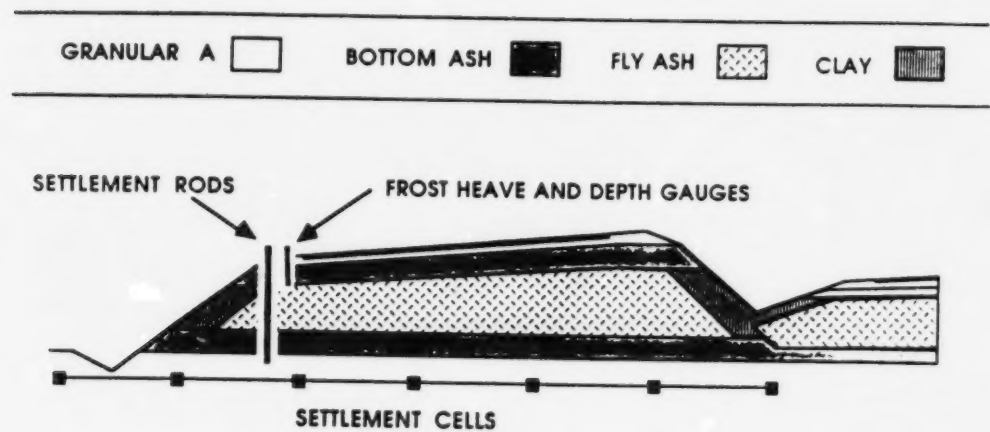
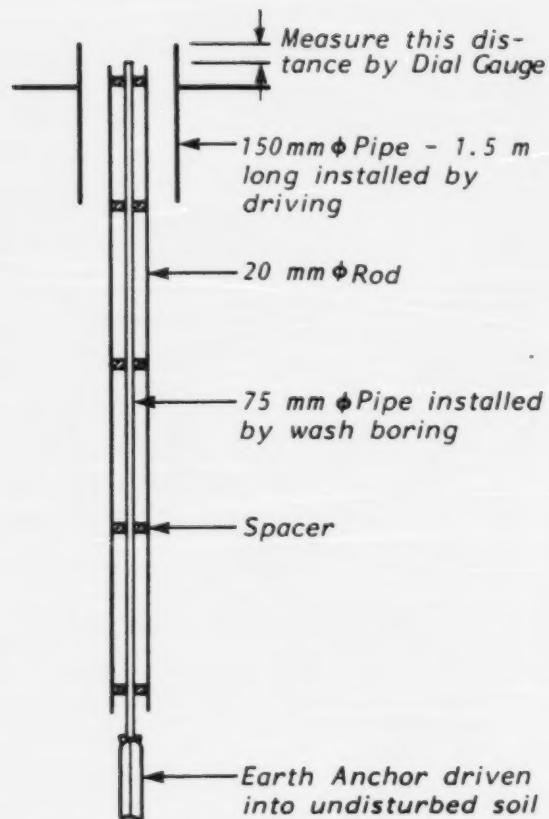


FIGURE 8
CROSS-SECTION OF HIGHWAY 402 COAL ASH EMBANKMENT



SARNIA EMBANKMENT

FIGURE 9
EARTH ANCHORED SETTLEMENT
GAUGE INSTALLATION

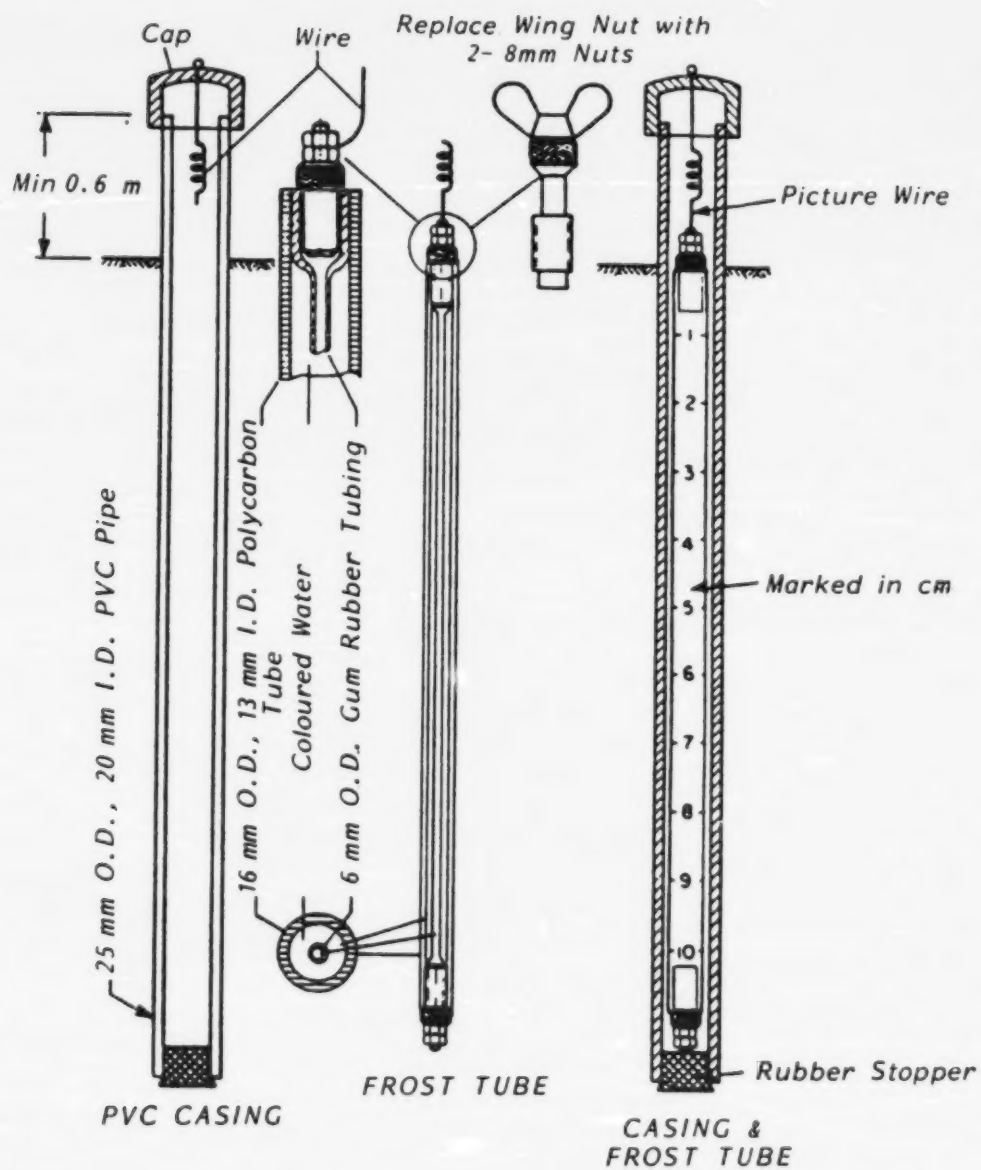


FIGURE 10
FROST DEPTH GAUGE

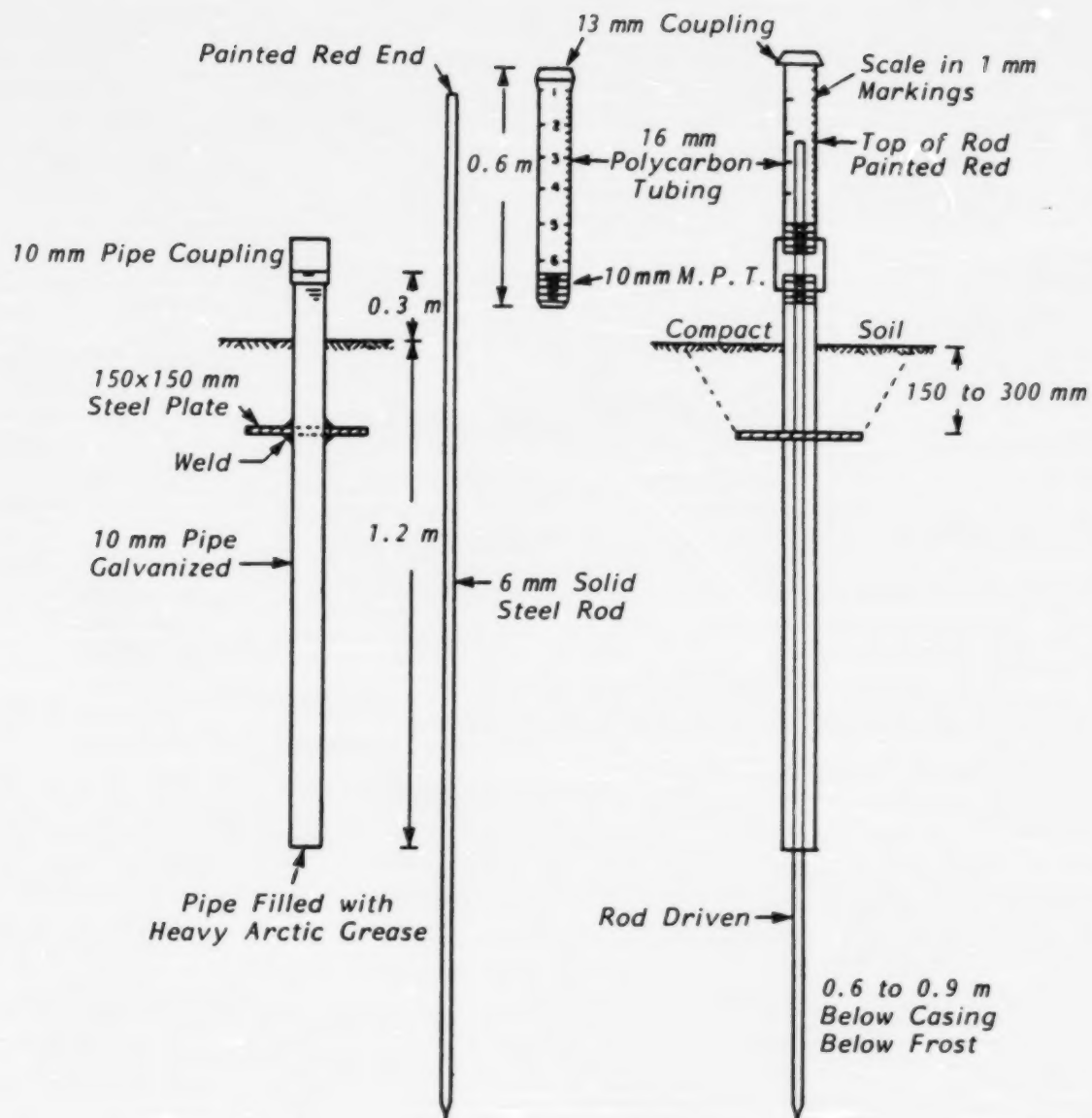


FIGURE 11
FROST HEAVE GAUGE

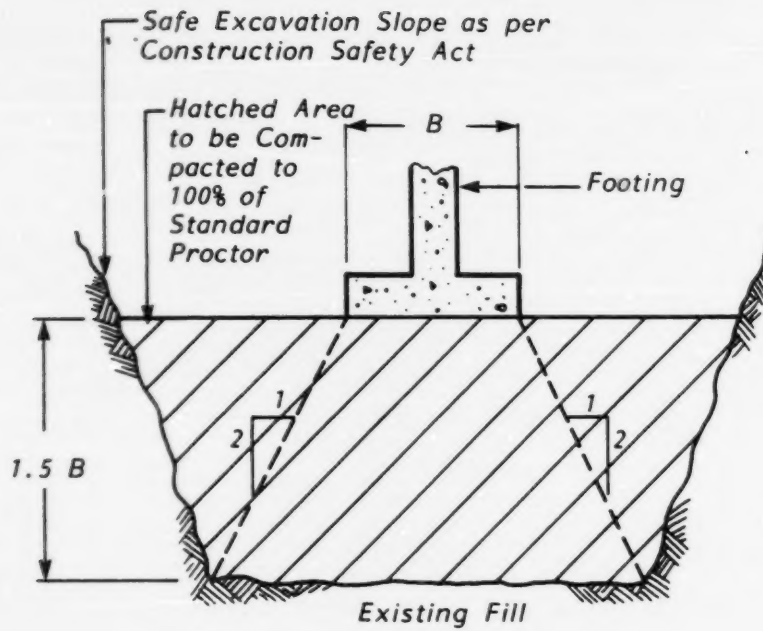


FIGURE 12
COMPACTION REQUIREMENTS UNDER FOOTINGS

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